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DISPLACEMENT-BASED DESIGN OF SDOF FRAMES CLADDED WITH DISSIPATIVE ROCKING PANELS

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Abstract

In this study, a simplified model is proposed to analyze the behavior of rocking panels with friction connections under seismic loads. Traditional rocking systems, often equipped with dissipative devices, present challenges for practitioners due to the complexity of the mathematical models involved, which require solving differential equations describing the negative tangent stiffness caused by the intrinsic instability of rocking motion.
The new model overcomes these difficulties by treating the system as a Single Degree of Freedom (SDOF) oscillator. This allows for the adaptation of displacement-based design (DBD) procedures without the need for complex closed-form solutions. The model simulates the behavior of rocking panels with friction connections, which offer substantial sliding stiffness and energy dissipation, providing both stability and damping to the structure. The proposed methodology is validated by comparing its predictions to pseudo-dynamic tests conducted on a single-storey precast industrial building, demonstrating excellent accuracy and reliability. The simplified approach enables practitioners to estimate the lateral response of these systems using a linear spectral analysis, offering a practical tool for the seismic design of structures that incorporate rocking panels and friction connections. Future developments will explore extensions of the model for multistorey buildings.

Keywords: rocking panels, friction connections, displacement-based design, self-centering structures, seismic design.

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1 INTRODUCTION

The performance of modern structures during seismic events is typically ensured by allowing controlled inelastic deformations in critical regions, such as plastic hinges at the base of columns and in beam-to-column connections. However, while this energy dissipation strategy is effective in preventing catastrophic collapse, it often results in significant residual deformations that compromise the structure's functionality. These deformations can lead to extensive and costly repairs, and in some cases, render the building unusable, requiring demolition. The economic impacts are further compounded by business interruptions and the social challenges of restoring earthquake-affected communities [1, 2].

To address these issues, earthquake engineering research has focused on reducing both lateral displacements and residual deformations through innovative design strategies. Over the past few decades, two main approaches have gained prominence: seismic base isolation and self-centering structures. Seismic base isolation, first developed in the mid-20th century, decouples the building's superstructure from its foundation using flexible isolation bearings. This significantly reduces the seismic demand on the superstructure, effectively isolating it from ground motion effects [3]. By allowing the base of the structure to move freely, the upper part experiences reduced forces, which helps to protect also the non-structural components [4]. However, this method can result in large displacements at the isolation level, posing challenges for utility connections and architectural detailing.

On the other hand, self-centering systems have emerged as a promising alternative, particularly for prefabricated structures and bridges [5, 6]. In self-centering systems, post-tensioned tendons, or other internal constraints allow structural components to rotate during seismic loading and then return to their original positions once the earthquake has subsided [7]. Other solutions provide the installation of self-centering dampers along-with external bracing systems [8]. The key advantage of these systems is their ability to reduce or eliminate residual deformations, thus ensuring that buildings remain functional with minimal or no repairs. Various innovative technologies, such as shape memory alloys and elastic springs, have been explored to further enhance this self-centering capability [9, 10].

In addition to base isolation and self-centering, rocking motion has been investigated as another means of improving the seismic performance of structures. Historically, many ancient buildings inadvertently exhibited rocking behavior during earthquakes due to their geometry and construction techniques. The study of rocking motion, first rigorously analyzed by Housner in 1963, demonstrated that larger, more massive structures tend to exhibit greater stability during seismic excitation due to the self-centering effects of gravity [11]. This uplift and rocking motion acts as a mechanical fuse, limiting the forces transmitted to the structure and allowing it to re-center naturally without the need for additional devices.

More recently, engineers have explored combining rocking systems with modern moment-resisting frames and dissipative devices to control displacements and enhance energy dissipation [1, 2, 9]. By incorporating friction connections, energy is dissipated during the rocking motion, further reducing the seismic demand on the structure. Despite the potential advantages, however, many of the models developed to simulate the behavior of rocking structures are highly complex and not easily applicable for practitioners [12]. To address this issue, this study proposes a simpler calculation model, adapted from the original Displacement-Based Design method [13], for predicting the seismic response of single-storey buildings implementing (cladding) rocking panels with friction connections, which is both practical and accurate for use in design applications.

2 ROCKING PANELS’ LATERAL STRENGHT

Recently proposed systems of rocking panels provide (all or a subset) these macro-components: (1) a main resisting frame (relevant parameters identified with the suffix “F”), made of reinforced concrete, steel or timber elements; (2) a group of rocking panels (“P”); (3) dissipative (friction or hysteretic) connections (“C”) linking each panel to the adjacent one or to the main frame. Despite the peculiarities, all the proposed layouts can be grouped in two categories:

- in the first one, depicted in Fig. 1-a, heavy concrete panels (P) are attached to the main frame (F) via a horizontal shear connector [14];
- in the second one, shown in Fig. 1-b and more suitable or light timber structures, CLT panels bear both horizontal and vertical loads [15].

In both systems, the rocking motion of the panels is enabled by shear keys (marked in blue in Fig. 1-a,b) at the base corners of the panels, which act as hinges alternately depending on the direction of movement. The upper connection of the panels, also shown in blue in Fig. 1-a,b, secures the top edge to the beam, facilitating the transfer of lateral seismic forces. In the second system, this same connection is used to transfer the gravitational load from the top slab as well, which results in increased lateral stiffness of the rocking panels. The overall lateral response of these rocking systems can be obtained through the effect-superimposition of the contribution of each macro-component: (1) main resisting frame (whose force-displacement response is qualitatively shown in Fig. 1-c); (2) bare rocking panel featuring a typical intrinsic negative tangent stiffness (see Fig. 1-d); (3) the dissipative connections highlighted in red in Fig. 1-a,b. In this study, friction connections linking the vertical edges of adjacent panels are considered. During the rocking motion, the relative vertical displacement between the panels generates energy dissipation within these elements (see Fig. 1-e). Moreover, the same feature a significant (positive) sliding/tangent stiffness that may compensate that (negative) of the bare panels. This interaction between the rocking panels and the friction connections allows the system to achieve both stability and energy dissipation, while maintaining a self-centering capability driven by gravity.

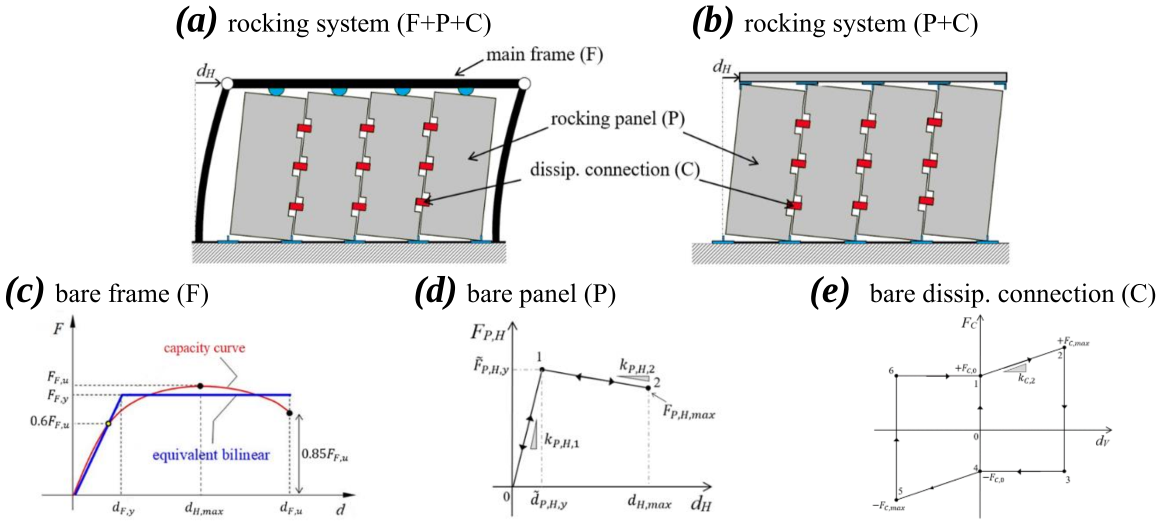

Figure 1: Rocking panels installed within a main resisting frame (a) or operating as a stand-alone system (b) and hysteretic response of each (macro) component: bare frame (c); bare panel (d); bare dissipative connection (e). Adapted from [16].

3 PROPOSED DBD CALCULATION METHOD

For sake of brevity, this section provides "only" a qualitative overview of the proposed Displacement-Based Design (DBD) method to predict the seismic response of single-storey frames featuring (cladding) rocking panels with friction connections. The model is designed to simulate the system's behavior as a single-degree-of-freedom (SDOF) oscillator, allowing engineers to use the displacement-based design (DBD) approach without the need for complex differential equation solutions. This makes the model suitable for practical applications where simplicity and efficiency are crucial.
In the proposed model, the overall lateral response of the system is described through a combination of three "spring-dashpot" systems (see Fig. 2-a), representing the main frame (F), the rocking panels (P), and the friction connections (C). Each component contributes to the system's overall stiffness and damping characteristics. The effective vibration period (T_{eff}) is calculated considering the overall secant stiffness (i.e., the sum of that offered by the frame, the panels, and the connections). Otherwise, the equivalent viscous damping (ξ_{eff}) is determined as the weighted average of the equivalent viscous damping developed by each component [13]. To simulate the response of the overall system under seismic loads, an (iterative) linear spectral analysis is employed (see Fig. 2-b). Since both the secant stiffness and the damping of each component depend on the displacement (unknown a priori), few iterations are usually needed to reach the convergence associated to the final/correct spectral displacement (i.e., $S_d^{(i)}(T_{eff}^{(i)}, \xi_{eff}^{(i)})$). An exhaustive description of this algorithm and the whole set of equations governing the dynamics of these systems can be found in a recent paper of the same authors [16].

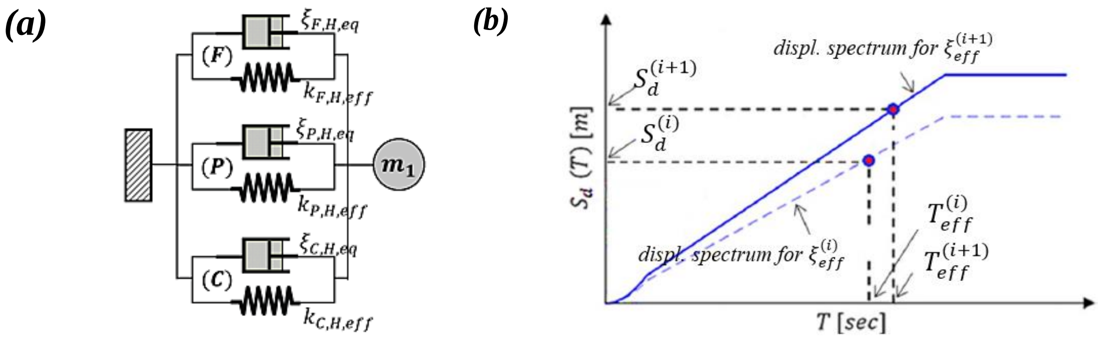

Figure 2: Adopted DBD calculation method: (a) mass-spring-dashpot model; (b) displacement response spectrum analysis. Adapted from [16].

4 EXPERIMENTAL VALIDATION

In this section, the proposed DBD calculation tool is validated using test results from a full-scale prototype of a precast industrial building equipped with dissipative rocking panels, installed according to Fig. 1-a. These tests were conducted at the European Laboratory for Structural Assessment (ELSA) facilities of the Joint Research Centre in Ispra, Italy [14]. Besides the quasi-static cyclic tests, also two pseudo-dynamic (PsD) tests were carried out to assess the accuracy of the (iterative) spectral procedure in predicting the building's lateral response under real seismic conditions. The prototype was a single-storey building, 8.13 m high, with two parallel frames 5.0 m apart in the transverse direction and 8.0 m bays in the longitudinal direction (see Fig. 3-a). Each frame had three square columns 50 x 50 cm inserted into pocket plinths fixed to the strong floor. Two foundation beams connected the frames and supported twelve rocking panels. Six columns carried roof beams (75 x 50 cm) supporting seven slabs (35 cm thick), with typical building masses for this type. The main frame was designed for seismic actions according to the EC8 response spectrum (see Fig. 3-b) for soil type B, with a Peak Ground Acceleration (PGA) of 0.36g at Ultimate Limit State (ULS). The original “Tolmezzo 1976” (Udine, Italy) ground motion record was slightly modified in order to ensure the spectral matching (see Fig. 3-b) with the EC8 design spectrum along with the whole range of periods (i.e., 0÷4.0s).

The rocking panels, 2.5m wide, 8.4m high and characterized by a cantilever lateral stiffness $k_{P,H,el} = 4.8kN/mm$, were equipped with: (i) steel "rocking sockets" at the base corners; (ii) ten single dissipative friction connection, i.e., one in between adjacent panels, featuring “yielding” strength $F_{C,0} = 60kN$, post-yield slope $k_{C,2} = 2.0kN/mm$; (iii) a top connector designed to transmit only horizontal loads (i.e., the main frame's gravitational load was supported by the columns, while the panels carried only their own weight). The fundamental period of the frame plus the bare rocking panels, i.e., devoid of dissipative connections, was measured to be *1.53s* with a participating mass m_1*=170 tons* (including the entire roof mass and 50% of the mass of the columns and panels).

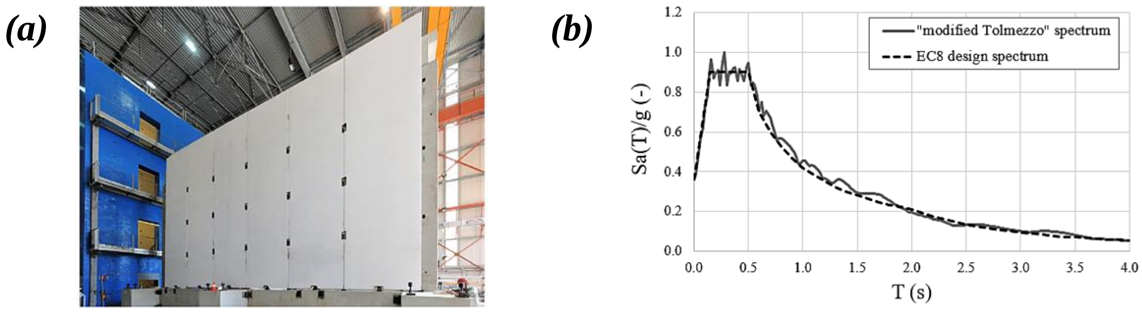

Figure 3: PsD tests: (a) picture of the arrangement of the dissipative rocking panels installed along longitudinal frame (adapted from [14]); (b) “Tolmezzo” ground motion record modified to fulfil the spectral matching with the EC8 design spectrum (adapted from [16]).

It is worth to noting that, during all the PsD tests, the main frame remained within the elastic range offering only a purely elastic viscous damping (about 5%) without any additional hysteretic energy dissipation due to the formation of plastic hinges. In particular, two different PsD tests were carried out:

- the first PsD test was performed at ULS level (i.e., *PGA=0.36g*): the predicted (absolute) spectral displacement is $S_d(T_{eff}, \xi_{eq}) = 51.0mm$ (see Fig. 4-a), against a peak of -57.1 mm measured experimentally (i.e., 10.7% of error, see Fig. 4-b), being $T_{eff} = 0.78s$ the effective oscillation period, and $\xi_{eq} = 16.8\%$ the overall equivalent viscous damping. It is worth noting that the addition of friction connections resulted in a reduced effective period but significantly increased damping and lateral stiffness;
- the second PsD test was executed at the SLS level (i.e., *PGA=0.18g*). In this test, the friction connections were removed, resulting in a structural system without significant energy dissipation (i.e., pure rocking panels) and leading to larger lateral deformations. Indeed, this configuration was not assessed at ULS to prevent damage to both the main frame (F) and the panels (P). The accuracy of the proposed analytical model is demonstrated in Fig. 5-b, which compares the peak displacement predicted by the spectral procedure with the experimental displacement recorded during the PsD test. The experimental peak displacement of 57.6 mm,

recorded at around t=6.3s, was only slightly higher (by 10.6%) than the predicted displacement of 52.1 mm.

In conclusion, these findings demonstrate that the proposed spectral procedure, while simple and sufficiently accurate, can be effectively used for the design of single-storey buildings with (cladding) rocking panels, whether equipped with or without dissipative friction connections. Future work will focus on extending this approach to multi-storey frames, including comparisons with NLTH analyses.

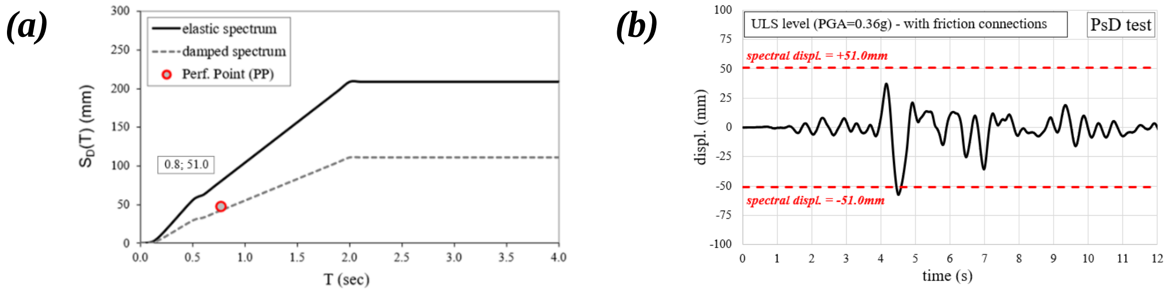

Figure 4: Seismic response of the case-study building at ULS level (*PGA=0.36g*): (a) maximum displacement predicted though the response spectrum analysis; (b) comparison with the displacement time-history recoded during the PsD test. Adapted from [16].

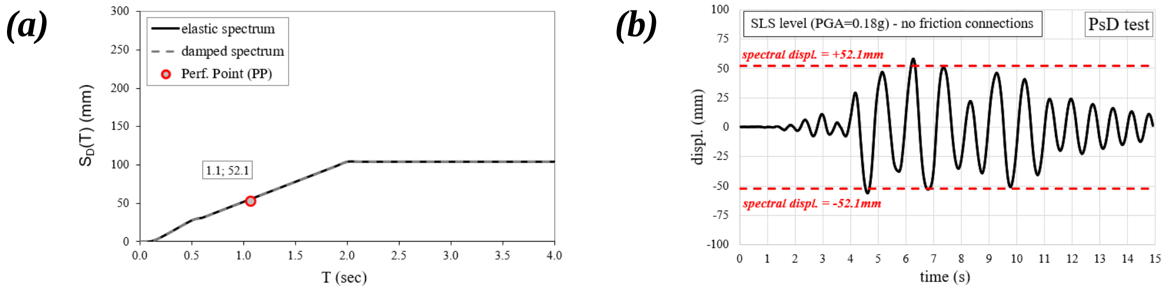

Figure 5: Seismic response of the case-study building at SLS level (*PGA=0.18g*): (a) maximum displacement predicted though the response spectrum analysis; (b) comparison with the displacement time-history recoded during the PsD test. Adapted from [16].

5 CONCLUSIONS

This paper presents a simple calculation tool, adapted from original DBD procedures, for the seismic design of single-story buildings featuring (cladding) rocking panels equipped or devoid of dissipative friction connections. The proposed model offers a practical solution for engineers, allowing them to estimate the lateral response of such systems using a simple (iterative) linear equivalent spectral analysis (i.e., avoiding to address more complex numerical simulations).

The tool was validated through the comparison with the results of pseudo-dynamic (PsD) tests carried out on a full-scale building prototype, demonstrating its accuracy in predicting the response during an intense seismic event at ULS (i.e., *PGA=0.36g*) and a minor one at SLS (i.e., *PGA=0.18g*). Indeed, in both cases, the predicted peak lateral displacements were affected by errors of less than 11%. Future developments of this study will focus on extending the model to multi-storey buildings and exploring the use of different types of dissipative connections, e.g., hysteretic and viscous dampers.

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